SUITABILITY OF USING CALIFORNIA BEARING RATIO TEST TO PREDICT RESILIENT MODULUS

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Abstract

Resilient modulus (M_r) of subgrade is a very important factor in airport and highway pavement design and evaluation process. Typically, this factor is evaluated using simple empirical relationships with CBR (California-bearing-ratio) values. This paper documents the current state of the knowledge on the suitability of this empirical approach. In addition, the paper also documents the use of finite element analyses techniques to determine the California Bearing Ratio. The stress-strain response of the various soils is simulated using an elasto-plastic model. The constitutive model employed is the classical von Mises strength criteria with linear elasticity assumed within the yield/strength surface. The finite element techniques employed are verified against available field and laboratory test data. The model is then utilized to predict the CBR of various soils. The empirical relationship between CBR and resilient modulus will then be investigated based on the results obtained from the three dimensional finite element analysis and its suitability for flexible pavement design will be evaluated.

Introduction

Most of the present methods used to design pavements utilize a mechanistic design procedure based on elastic layer theory (Asphalt Institute, 1982; Shell, 1977; and FAA, 1995). The elastic modulus for the soil subgrade can be obtained from repeated load triaxial tests (AASHTO 1993). Due to the complexity of the testing and test equipment required for the repeated load triaxial tests, it is desirable to develop approximate methods for the estimation of resilient modulus. The AASHTO design guide suggests that the resilient modulus of fine-grained soils can be estimated as (Heukelom and Klomp 1962):

$$M_r \text{ (psi)} = 1{,}500\ CBR \qquad (1)$$

In addition, there are various other relationships that are used around the world:
U.S. Army Corps of Engineers (Green and Hall 1975)

$$M_r \text{ (psi)} = 5{,}409\ CBR^{0.71} \qquad (2)$$

South African Council on Scientific and Industrial Research (CSIR)

$$M_r \text{ (psi)} = 3{,}000\ CBR^{0.65} \qquad (3)$$

Transportation and Road Research Laboratory (TRRL)

$$M_r \text{ (psi)} = 2{,}555\ CBR^{0.64} \qquad (4)$$

There has been considerable discussion on the suitability of using any of these approaches. The CBR (California Bearing Ratio) test is a measure of the shear strength of the material and does not necessarily correlate with a measure of stiffness or modulus such as the M_r. Thompson and Robnett (1979) could not find a suitable correlation between CBR and resilient modulus. In addition, it is also known that the resilient modulus is dependent on the applied stress level (Rada and Witczak 1981). For most fine-grained subgrade soils, M_r decreases with increasing deviatoric stress level. Model forms characterizing the relationship between M_r and deviatoric stress have been shown to be bi-linear, hyperbolic, semilog and log-log (Witczak et al. 1995).

The CBR test can be thought of as a bearing capacity problem in miniature, in which the standard plunger acts as a circular footing. Using the bearing capacity equation, CBR was correlated with the undrained shear strength, s_u as:

$$CBR = 0.62\ s_u\ (psi) \quad (5)$$

Black (1961) found satisfactory correlation with the above value. In addition it was also shown by Duncan and Buchignani (1976) that the resilient modulus can be predicted using the undrained shear strength knowing the plasticity index (PI) of the soil.

$$M_r = 100 – 500\ s_u \qquad PI>30$$
$$M_r = 500 - 1500\ s_u \qquad PI<30 \quad (6)$$

Combining equations (5) and (6),

$$M_r\ (psi) = 160\ \text{to}\ 2420\ CBR \quad (7)$$

Thompson and Robnett (1979) suggested a relationship utilizing the unconfined compressive strength, Q_u to determine M_r.

$$M_r(ksi) = 0.307 Q_u(psi) + 0.86 \quad (8)$$

From equation (7) and (8), it can be seen that there is a wide variation in the resilient modulus value that can be obtained using the CBR depending on the plasticity properties of the soil. In this study, the suitability of using equation (1) in the AASHTO and FAA design code will be discussed. In addition, the use of three-dimensional finite element models utilizing plasticity models will be used to predict CBR values. This study is a precursor to further studies utilizing three-dimensional finite element models with plasticity parameters to predict the performance and failure mechanisms of flexible pavement systems.

Some Background on Finite Element Analysis

An objective of this paper is to demonstrate that readily available displacement based and hybrid (combined stress and displacement solution variables) based finite elements formulations are capable of accurately, and efficiently calculating the California Bearing Ratio of subgrade soils and thereafter the performance of pavement systems. Available displacement based and hybrid (combined stress and displacement solution variables) based finite elements formulations are capable of accurately and efficiently calculating limit loads for pavement systems. An important feature in the successful use of displacement based finite element formulations is the use of reduced integration techniques in many limit analysis investigations. The term ‘reduced’ integration refers to the fact that a lower level (fewer sampling points) of numerical integration is being used than that theoretically required, to exactly integrate a polynomial of a certain order.

Alternatives to the use of reduced integration exist, e.g. hybrid finite elements, or very high order displacement-based elements such as the 15-noded cubic strain triangle. Hybrid elements are available in commercial codes, such as ABAQUS (2000), and are effective in the analysis of incompressible materials. The term hybrid stems from the use of both displacement and stress components as solution variables. In this case, the stress component included is the mean pressure. Zienkiewicz and Taylor (1994) and HKS (2000) discuss this in detail. More discussion about the suitability of these elements and analysis techniques can be found in Sukumaran et al. (1998).

Verification of Finite Element Modeling Techniques

The adequacy of finite element modeling utilizing plasticity models are demonstrated in the following by virtue of their performance in accurately calculating the California Bearing Ratio for a subgrade soil. The subgrade soil utilized for the modeling purpose is the medium strength subgrade used in the construction of the pavement test facility at the FAA technical center. Three verification studies were conducted. The first one utilized the ultimate shear strength as the yield strength. The properties of the soil used are shown in Table 1.

Table 1: Properties of Medium Strength Subgrade Soil

Soil Property	Values
Moisture content	30.5%
Undrained shear strength	13.3 psi
Dry density	90.5 pcf
Elastic modulus	12,000 psi

The finite element mesh used for the analysis is shown below in Figure 1. The finite element analyses were conducted using ABAQUS (HKS 2000). A von Mises shear strength idealization was used to model the clay. The elastic-plastic material properties used for the soil are shown in Table 1. The von Mises model implies a purely cohesive (pressure independent) soil strength

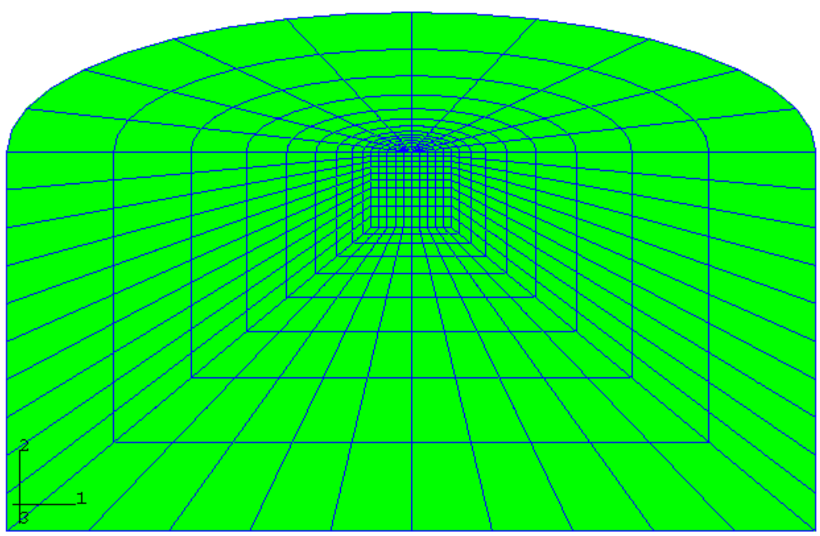
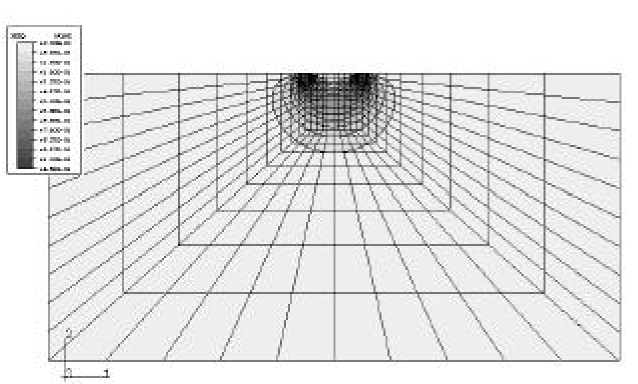


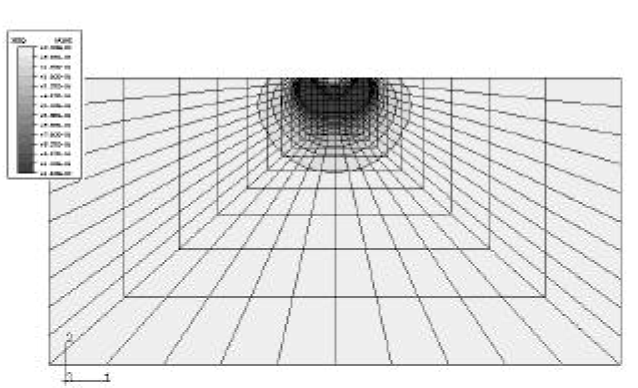
Figure 1. Finite element mesh used in the analysis

definition. A three dimensional response was simulated using quasi three-dimensional Fourier analysis elements (CAXA) available within ABAQUS. CAXA elements are biquadratic, Fourier quadrilateral elements. The number of elements and nodes in the mesh are 185 and 6260 respectively.

The second study was conducted using the von-Mises model with unconfined compression stress-strain data. Stress-strain response can be better captured if stress vs. strain data from unconfined compression tests, triaxial tests or direct simple shear test are input to obtain the plasticity model parameters. It can be seen from Figure 2 that the zone of plastic strain increases as penetration depth increases as would be expected. The third study conducted utilized the instantaneous elastic modulus, which was calculated from the unconfined compression stress-strain data. Table 2 summarizes the results obtained. It can be seen that the von-Mises model utilizing the ultimate shear strength input predicts CBR values that are closer to the higher end of the measured CBR values, while the other two cases predict values closer to the lower end of the CBR values measured. Several analyses were also conducted using linear elastic models utilizing elastic modulus values predicted using Equations (1) to (4). All these analyses rendered very high values of CBR.



(a)



(b)

Figure 2. Plastic strain distribution at a) 0.1” piston penetration (b) 0.2” piston penetration

Table 2: Results of the Finite Element Verification Studies on the Medium Strength Subgrade

Finite Element Model Utilized	CBR values computed
Von-Mises with ultimate shear strength input (Analysis 1)	CBR at 0.1?= 8.6 CBR at 0.2?= 5.7
Von-Mises with stress-strain data input (Analysis 2)	CBR at 0.1?= 5.6 CBR at 0.2?= 4.8
Elastic model utilizing stress-dependent elastic modulus (Analysis 3)	CBR at 0.1?= 4.2 CBR at 0.2?= 4.1
Field measurements (NAPTF test pits, November 1999)	CBR at 0.1?= 3.4-8.4 CBR at 0.2?= 2.8-7.2

In order to understand the stress-strain response of the soil, stress vs. displacement plots were studied for the three cases mentioned above and compared with the field test data. The stress-strain plots are shown in Figure 2.

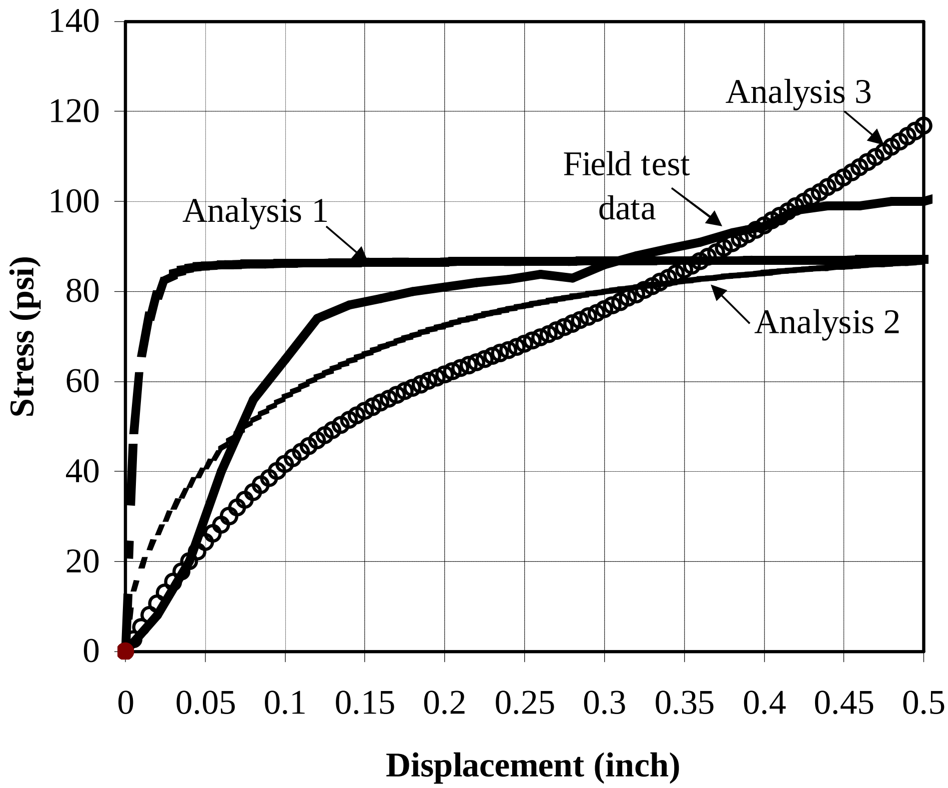


Figure 3: Stress vs. displacement plot for the various verification studies compared with field test data

The load vs. displacement response computed shows a remarkable similarity to what was observed in the field. The prediction of the CBR value also improves as a consequence. From the results, it can be seen that three-dimensional finite element modeling can accurately capture the stress-strain response of the subgrade soil. Based on this conclusion, it was decided to model various other soils for which measured resilient modulus and unconfined compressive strength data existed (Drumm *et al*. 1990).

Relationship Between CBR and Resilient Modulus

The data provided by Drumm et al. (1990) was for 11 subgrade soils from Tennessee, which had clay contents ranging from 16 to 55%. The soil properties of interest are summarized in Table 3. Additional soil properties are given in Drumm *et al*. (1990).

Table 3: Index Properties of Soil Tested by Drumm *et al*. (1990)

Soil Classification			**Clay content (%)**	**Atterberg Limits**			**Unconfined compressive strength (psi)**	**Breakpoint resilient modulus (psi)**
Designation	USCS	AASHTO		LL	PL	PI		
A31	CL	A-4	17	30.5	22.1	8.4	63.3	15,000
B21	CL	A-6	18	38.8	23.3	15.5	68.8	14,000
C11	SM	A-2-4	17	20.7	19.0	1.7	30.9	11,500
D11	ML	A-4	18	36.2	34.1	2.1	28.7	2,000
E21	ML	A-7-6	35	37.1	27.0	10.0	67.7	18,000
E31	CL	A-4	36	42.1	22.0	20.1	45.6	8,000
F11	CL	A-7-6	16	29.5	20.1	9.4	53.5	6,000
H11	CL	A-4	20	28.5	19.2	9.3	62.6	7,500
H21	SM-CL	A-4	16	21.0	14.1	6.9	39.7	8,000
J11	MH	A-7-5	28.7	68.5	39.2	29.3	27.3	12,000
J31	MH	A-7-5	55	69.5	42.6	26.9	46.0	17,000

CBR values were predicted for these soils using the elasto-plastic von-Mises model and the finite element mesh shown in Figure 1. The soil properties used in the model are as listed in Table 3. The unconfined compressive strength was input as the yield strength. The CBR values computed for the various soils are listed in Table 4.

Figure 4 shows the comparison between the measured resilient modulus values and the values predicted utilizing the computed CBR values and equations (1) to (4). In addition, the resilient modulus was also predicted utilizing the unconfined compressive strength and equation (8). It can be seen that equations (1) to (4) over predict the resilient modulus by a factor of 2 or more. The best estimate of the resilient modulus is obtained from equation (8) suggested by Thompson and Robnett (1979).

Table 4: Predicted Values of CBR from Finite Element Analyses

Soil Designation	CBR values predicted from FEA
A31	40.4
B21	38.84
C11	19.3
D11	11.0
E21	40.4
E31	24.8
F11	25.3
H11	30.1
H21	21.71
J11	17.01
J31	28.61

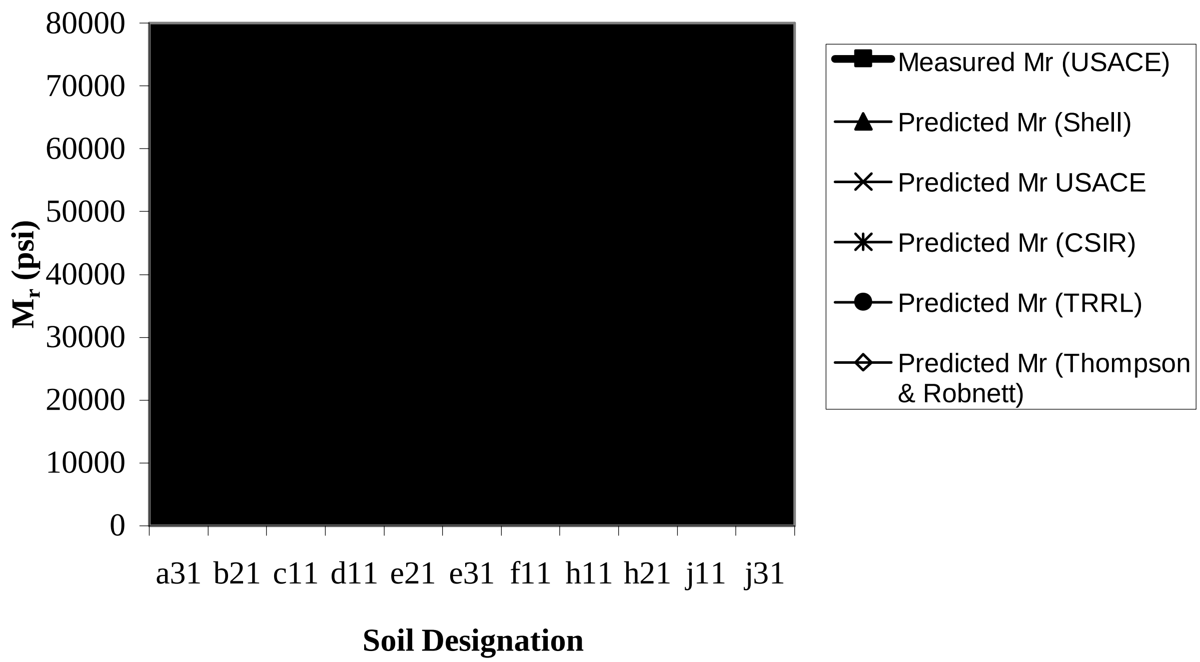


Figure 4: Comparison between the measured and predicted resilient modulus values

Conclusions

Mechanistic design methods utilizing elastic layer theories require the determination of the elastic moduli. The elastic moduli for soil subgrades can be characterized by the resilient modulus and can be obtained from the repeated load tests. Due to the time and skill required to conduct these tests, approximate correlations between resilient modulus and some more easily measured parameter is utilized. The commonly used California Bearing test value is used to obtain a prediction of resilient modulus. During the course of this research, it was found that the resilient modulus values could not be suitably predicted using Equation (1). It was observed during the present research that the relationship given by Equation (1) overpredicts the resilient modulus. A more suitable estimate of resilient modulus can be obtained from Equation (8) knowing the unconfined compressive strength of the soil.

Plasticity models should be utilized when realistic evaluations of strains and displacements are required. Elastic models, especially the Duncan hyperbolic model (Duncan and Chang 1970) can suitably predict deformations at failure as long as the orientation of stresses remain constant but have limited benefit when evaluating displacements at and after failure. In addition, the hyperbolic model is of limited suitability if realistic evaluations of pore pressure are required. Linear elastic models are of limited benefit as they do not accurately predict stresses or strains in the subgrade soil.

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